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WAVE-INDUCED RUNUP AND OVERTOPPING TRANSMISSION FOR CORE-LOC™ ARMOR LAYERS

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Ivano Melito[1] and Jeffrey A. Melby[2 §]

Abstract

Wave-induced runup and wave transmission due to overtopping are two critical variables to be considered in design of coastal structures. Many studies have been performed on different armor units, but there is no generalized information available on CORE-LOC™ armor layer performance with respect to wave runup and transmission. Thus, an experimental study was performed to investigate the runup and transmission response of a CORE-LOC™ armor layer. Wave runup and transmitted wave heights were measured for a wide range of wave, water level, and structure conditions in order to develop predictive tools for CORE-LOC™ layer response. A new empirical model to predict runup levels was developed from an existing model for rock revetments. Furthermore, a semi-quantitative analysis of transmission data was conducted to give insight and guidance for design purposes.

1. Introduction

Wave-induced runup levels and transmission due to wave overtopping are important parameters to design the crest height of coastal structures. Different types of armor units, including rock, dolos, tribar and tetrapod were investigated to determine wave runup, overtopping and wave reflection as well, but generalized studies of CORE-LOC™, hereafter referred to as Core-Loc, armor layer wave runup and transmission due to overtopping have not been conducted yet. Therefore, the goal of this study was to measure simultaneously, on a rubble mound structure armored with Core-Loc, wave runup levels and transmitted wave height due to overtopping for different wave conditions, spectral shape, water levels, and structure permeability.

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The complete data analysis and test results will provide guidance to improve the initial design of coastal structures with Core-Loc armor layers.

2. Test Setup

An extensive series of small-scale physical model tests were conducted at the U.S. Army Engineer Research and Development Center, Waterways Experiment Station in Vicksburg, Mississippi. All tests were performed in a 4-channel wave flume measuring 74.6 m long, 3.0 m wide, and 2.0 m high, with the test structure section installed in the first external channel (0.9 m wide), about 55.7 m from the wave generator board (Figure 1).

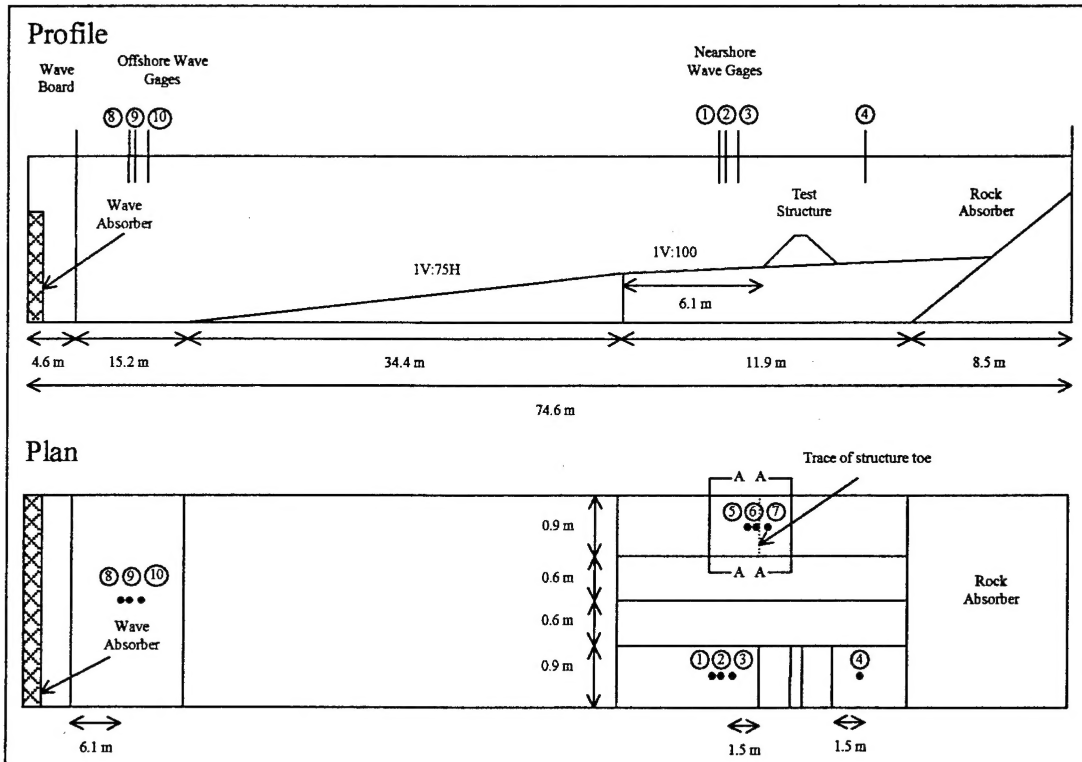

Figure 1 Flume profile and plan

The approach slope was 1V:100H for 6.1 m seaward of the structure toe, 1V:75H for 34.4 m, and flat for 15.2 m to the generator pit. Three different water levels were used during the tests: $h_s = 0.2$ m, a depth limited condition with severe wave breaking in most tests, 0.5 m, where only the highest waves were breaking on the approach slope, and 0.7 m, where the wave were non-breaking. Here h_s is the water depth at the structure toe. The crest height was such that a water depth of $h_s = 0.5$ m provided a crest freeboard of 0.3 m that corresponded to an elevation of approximately one significant wave height above the still water level for the larger wave conditions tested. This is typical of prototype structure crest heights relative to prototype wave heights.

The structure cross section is shown in Figure 2. The armor layer was built using Core-Loc armor units with a mass of $M = 272$ g, a characteristic length of $C = 8.1$ cm and a specific gravity of $S = 2.3$ g/cm^3. The Core-Loc armor layer was placed in a single thickness at an average packing density coefficient of $\phi = 0.60$, as recommended by Turk and Melby (1997). The packing density coefficient is defined as $\phi = nk_\Delta(1\text{-}P)$, where $n = 1$ is the number of layers, $k_\Delta = 1.51$ is the empirical layer coefficient, and $P = 0.6$ is the porosity of the armor layer. The placement of Core-Loc model armor units followed WES guidelines for random hand placement in the laboratory: they were lowered vertically into position, placed on the slope rather than dropped, and placed so that they touched their neighbors. Moreover, units could not be pushed into a hole and particular orientation of units or placement patterns were avoided. The stone underlayer was sieve-sized, passing 3.17-cm and retained on 2.00-cm sieves. The underlayer thickness was 4.4 cm. The core material was sieve-sized, passing 0.93-cm sieves, which provided low permeability. A medium permeability was also tested, building the core with the same material used for the underlayer. The crest width was set to the minimum for overtopping conditions, three times C, as recommended in the Shore Protection Manual (1984). The seaward structure slope was 1V:1.5H, as Core-Loc units have been designed to be placed in a single-unit-thickness layer on steep or shallow slopes (Melby and Turk 1997). All test runs were limited to 15 min to minimize buildup of reflected wave energy in the wave flume. A rock absorber was placed behind the structure to absorb transmitted wave energy.

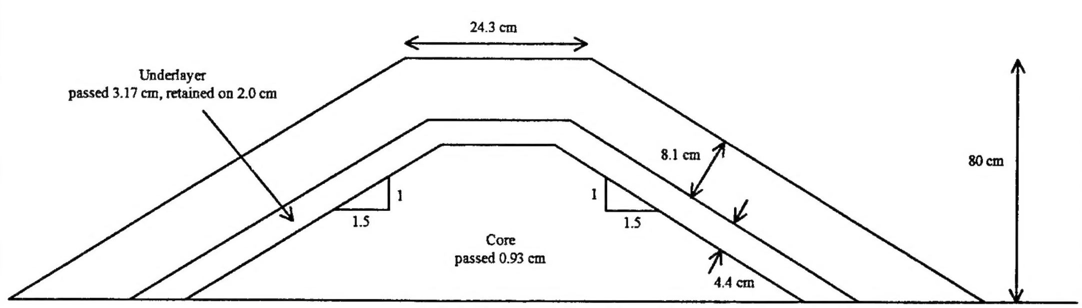

Figure 2 Model structure cross section

Irregular waves were generated based on the Texel, Marsen, and Arsloe (TMA) spectrum (Bouws et al. 1985), a finite depth variant on the JONSWAP spectrum. A piston-type wave board powered by an electro-hydraulic pump controlled by a computer-generated signal was used to generate waves. The wave board was commanded at a rate of 20 Hz, and ramps of 10 sec each were placed at the beginning and end of the command time series to gradually start and stop the wave generator. Signals for the wave generator were created to produce waves larger than would be needed, then run at a range of signal gains to produce varying wave heights. Waves were measured at a rate of 20 Hz using capacitance-type wave gages. The gages were calibrated with an automated technique. The deep wave gage array was positioned 6.1 m from the wave generator, and the shallow arrays positioned 1.5

m seaward of the structure toe and at the structure toe. For the three arrays of three gages each, the distance between gages was as shown in Figure 3. A single wave gage was placed on the rear of the structure, 1.5 m from the structure (landward) toe, as shown in Figure 1, to measure transmitted waves. Wave gage layouts for the two nearshore arrays are shown in Figures 3 and 4. For the largest waves measured herein, this gage location nearly corresponded to the recommendation by Goda (1985), where the design breaking wave is determined $5H_s$ seaward of the toe; this is the travel distance of large breaking waves. Incident and reflected waves were resolved using the National Research Council of Canada's (NRC) GEDAP analysis package. GEDAP uses time series from the three-gage arrays and the technique of Mansard and Funke (1987). Time series parameters were determined using a zero-downcrossing method.

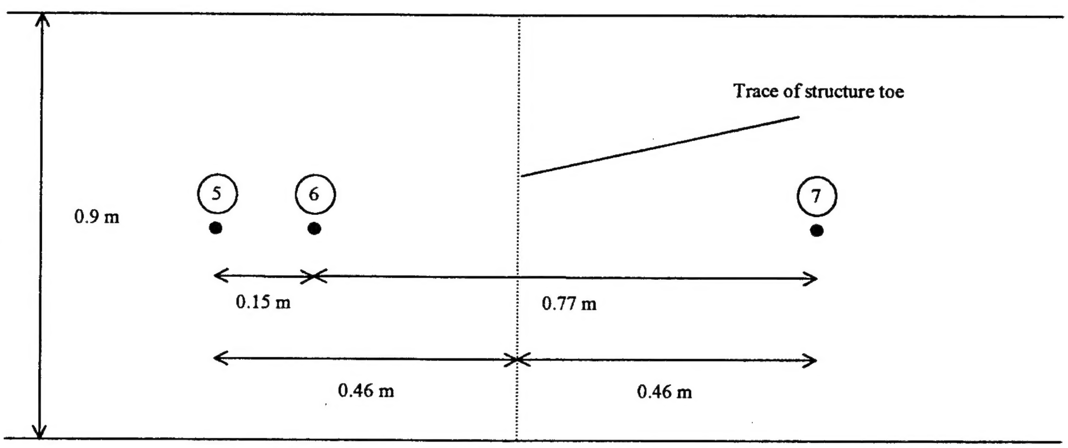

Figure 3 A-A plan and distance between the three wave gages

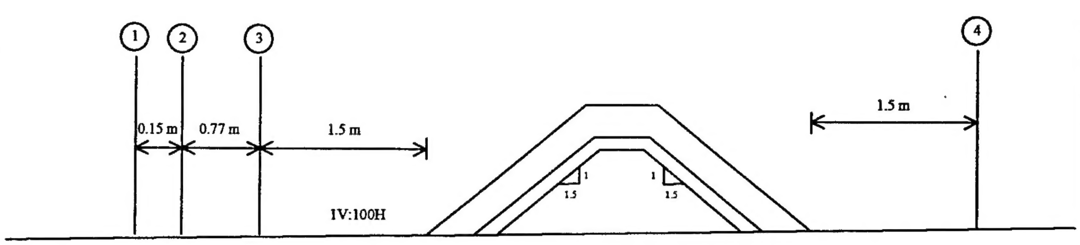

Figure 4 Structure with nearshore wave gage layout

Traditionally, the incident waves are measured without the structure in place (Hughes 1993). For this study, the waves in the flume were measured simultaneously with and without the structure in place. For the condition with no structure in place, an array of three gages was placed in the second channel of the flume, with its centroid at the location of the structure toe, as shown in Figures 1 and 3. Wave heights measured without the structure in place were smaller than those measured with the structure in place. This difference between the two sets of measurements was partially due to the fact that the wave generation system did not allow absorption of reflected and re-reflected waves within the flume, so wave energy built up in the flume during testing. It should be noted that systems applied to absorb reflected energy are not completely effective, so it is virtually impossible to prevent wave energy increase due to reflected waves.

A capacitance wire was stretched along the slope of the structure, just above the Core-Loc armor layer, to measure the water level oscillation along the structure slope. These runup elevations were vertically measured with respect to the still water level by a down-crossing method. On the basis of the study of Van der Meer and Stam (1992), various runup level statistics were chosen from the measured runup distribution: the maximum, 1%, 2%, 5%, 10% exceedance values, the significant (average of the highest one-third of the runup heights), and the mean runup. The percentage of exceedance was computed based on the number of the individual runup elevations in the time series. The maximum runup elevation was considered with less emphasis than the other statistics because it was highly variable, depending to a large extent on the wave realization.

The nearshore incident-wave characteristics used in the following analysis were those measured without the structure in place. The analysis was conducted separately for frequency domain and time domain parameters, both measured at the location of gage 6 shown in Figure 3. The relative runup for the x^{th} level is given by Ru_x/H, where H is the incident significant wave height at the toe of the structure (at gage 6, as mentioned above). Following a common approach in runup studies, the relative runup levels were described as function of the Iribarren parameter or surf similarity parameter (Battjes 1974) defined as

$$\xi = \tan\alpha / s^{1/2} \qquad (1)$$

where $s = H/L_o$, H is either H_{mo} for frequency domain analysis or $H_{1/3}$ for time domain analysis, $L_o = gT^2/2\pi$ with T is either T_p for frequency domain analysis or T_m for time domain analysis, α is the angle between structure slope angle and the horizontal, and g is the acceleration of gravity. This distinction for H_s should prevent any confusion between H_{mo} and $H_{1/3}$, as highlighted originally by Thompson and Vincent (1984), and more recently by Hamm and Peronnard (1997). The wave steepness s is fictitious as the nearshore wave height at the toe of the structure is related to the wave length in deep water.

Various series of tests were performed for this study. Each series contained five wave periods with four or five different wave heights for each wave period. Two different spectral shapes were tested using a spectral peakedness of $\gamma = 3.3$ or 7. For a water depth of $h_s = 0.7$ m, only the wave transmission due to overtopping was measured, runup was not measured. The test program for runup level measurements and transmitted wave height due to overtopping along with the range of parameters tested is shown in Table 1; the total number of tests performed was 123.

The analysis of runup levels on Core-Loc armor slopes is treated separately from the analysis of transmitted wave height in the following.

Table 1
Summary of Test Conditions

Series	h_s	γ	P (Core)	No. of tests	ξ_p	ξ_m	H_{mo}/h_s	$H_{1/3}/h_s$
1	0.5 m	3.3	Low	21	3.0 - 7.9	3.0 - 6.2	0.07 - 0.40	0.07 - 0.42
2	0.5 m	7	Low	21	3.2 - 7.8	3.0 - 6.3	0.07 - 0.42	0.07 - 0.44
3	0.2 m	3.3	Low	20	3.2 - 10.7	3.2 - 6.2	0.31 - 0.57	0.32 - 0.62
4	0.2 m	7	Low	4	6.7 - 8.7	4.3 - 6.2	0.33 - 0.54	0.36 - 0.59
5	0.7 m	3.3	Low	21	3.0 - 7.6	2.9 - 6.1	0.05 - 0.32	0.05 - 0.32
6	0.7 m	7	Low	4	4.4 - 6.6	3.7 - 5.6	0.15 - 0.33	0.15 - 0.34
7	0.2 m	3.3	Medium	10	3.7 - 9.3	3.2 - 5.8	0.33 - 0.59	0.33 - 0.64
8	0.5 m	3.3	Medium	11	3.0 - 7.9	2.9 - 6.2	0.09 - 0.41	0.08 - 0.43
9	0.7 m	3.3	Medium	11	3.4 - 7.4	3.0 - 6.0	0.05 - 0.32	0.05 - 0.32

3. Analysis of runup levels

As mentioned above, various runup level statistics were computed from the measured runup distribution. The maximum runup was analyzed, but will not be discussed in this analysis for the reasons explained above. For this portion of the study, the surf similarity parameter includes the effects of wave height and wave period only, as the structure slope was not varied during the tests. Plots of relative runup levels versus the Iribarren parameter are presented below to analyze the effect of water depth, spectral peakedness, and core permeability.

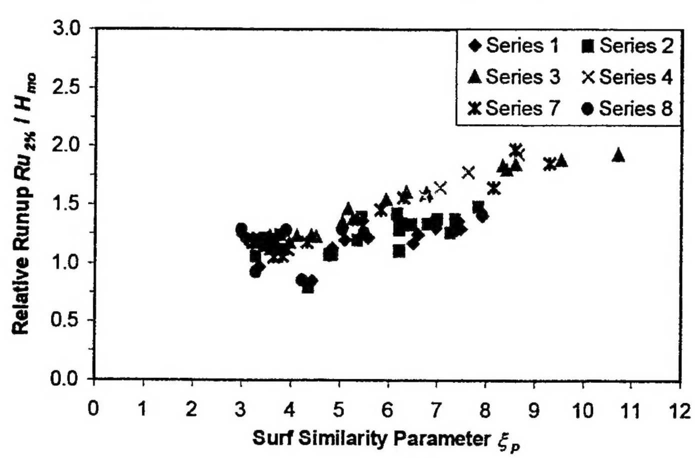

Figure 5 $Ru_{2\%}/H_{mo}$ versus ξ_p

The focus is on the 2% relative runup level, $Ru_{2\%}/H_{mo}$, versus ξ_p. In general, similar trends were found for the other relative runup statistics, and for $Ru_x/H_{1/3}$ versus ξ_m. However, a larger scatter in the data was noted in plots of $Ru_x/H_{1/3}$ versus ξ_m. This is mainly due to the fact that T_p is more stable than T_m, and is less susceptible to distortion by measurements or calculation errors (Allsop, Durand, and Hurdle 1998). All the test results for the 2% relative runup level versus ξ_p and ξ_m are shown in Figures 5 and 6, respectively.

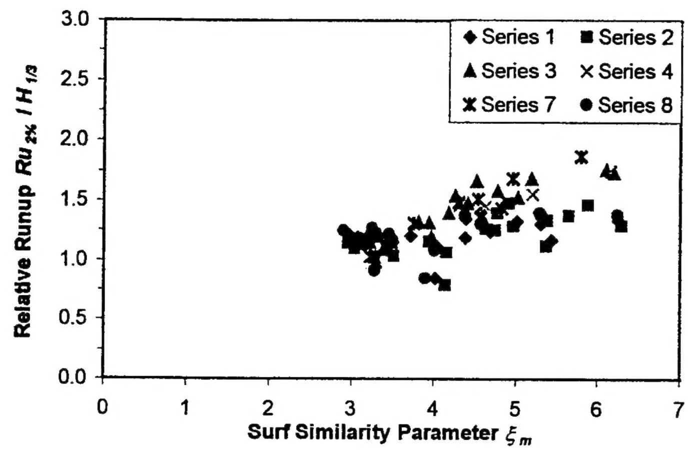

Figure 6 $Ru_{2\%}/H_{1/3}$ versus ξ_m

3.1 Effects of water depth

In this study, depth limited tests were performed with a water depth of $h_s = 0.2$ m which produced severe wave breaking through most of the 15-min tests, and $h_s = 0.5$ m where wave breaking occurred on the approach slope only for the highest waves. Figure 7 shows the relative runup $Ru_{2\%}/H_{mo}$ versus the surf similarity parameter ξ_p for the two depths considered.

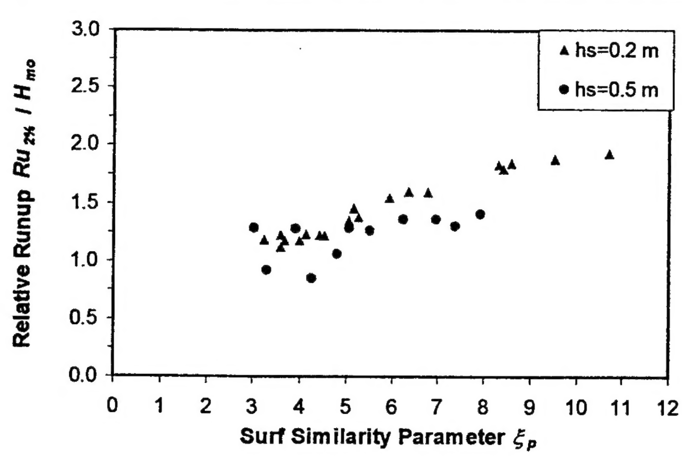

Figure 7 Effects of water depth

In general, for a given value of the surf similarity parameter, the relative runup increases for decreasing water depth. This is in contrast to the result found by Van der Meer and Stam (1992) for rock slopes, and by Van der Meer and Janssen (1994) for revetments, but consistent with the work of Rathbun et al. (1998) for an impermeable revetment. As for the latter study, at the shallower water depth, where wave breaking occurred over the sloping foreshore, a certain percentage of the incoming waves reached the structure as white water bores rather than green water waves. This meant the wave shape at the toe of the structure was affected by the sloping foreshore and produced a significant effect on the runup. It should be noted that Van der Meer and Janssen (1994) suggested a reduction factor to take in account the effects of a shallow foreshore. This factor was based on the fact that wave heights at the structure toe are no Rayleigh distributed when they break on the approach slope before to reach the structure. Moreover, Van der Meer and Stam (1992) referred to tests performed mostly in relatively deep water in front of the structure with very few tests under depth-limited conditions.

3.2 Effect of spectral peakedness

Most tests were undertaken using a spectral peakedness of $\gamma = 3.3$; however, a significant number of tests (29 in total) were performed with $\gamma = 7$ to investigate the effects of spectral peakedness. Results are shown in Figure 8. It can be seen that the effects of spectral peakedness on relative runup are negligible.

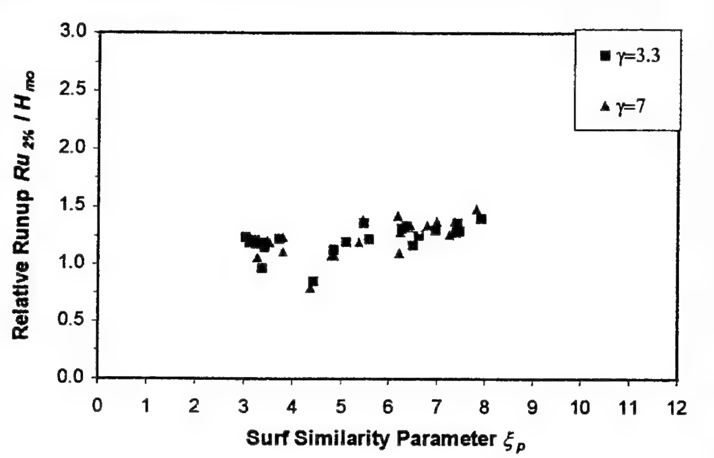

Figure 8 Effects of peakedness

3.3 Effects of core permeability

As mentioned earlier in this paper, two different core permeability levels were used during the tests. Low permeability was achieved using a sieve sized stone underlayer 4.4-cm thick,

passing 3.17-cm and retained on 2.00-cm sieves, and a core material passing on 0.93-cm sieves. A medium permeability was achieved building the core with the same material used for the underlayer.

Figure 9 contains data for tests performed with these two permeability levels. It can be seen in Figure 9 that a medium permeability level allows a slight reduction in relative runup due to the fact that wave up-rush flows into the more porous structure rather than over the surface. In addition, the more porous structure probably dissipates more of the energy in the up-rushing wave.

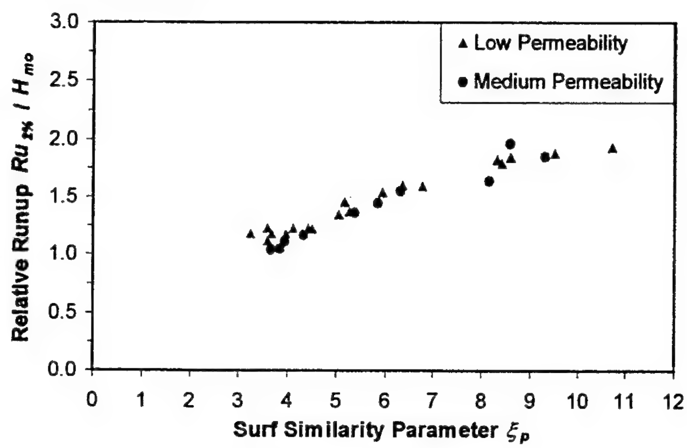

Figure 9 Effects of core permeability

4. Prediction method

An empirical method was sought to predict relative runup levels related to the surf similarity parameter. The prediction method was developed from the model proposed by Ahrens and Heimbaugh (1988) given as

$$Ru_{max}/H_{mo}=a\xi_p/(1+b\xi_p) \qquad (2)$$

This model resulted more accurate if the local wavelenght, found using linear wave theory, was used in the surf similarity parameter. Moreover, Ru_{max} values used in the model were based on tests with duration of 256 sec.

On the basis of the Ahrens and Heimbaugh's model, Rathbun et al. (1998) developed a predictive model for $Ru_{2\%}/H_{mo}$ accounting for the effects of depth-limited conditions on wave runup by the means of an enhancement factor. They used the ratio of the incident significant wave height to the water depth (both at the structure toe), H_{mo}/h_s, to define if breaking condition exists at the structure toe.

An alternative approach was followed in this study to fit the results for the relative runup level statistics with a revised empirical method. The model used is given as

$$Ru_x/H= a\xi/(1+b\xi) \qquad (3)$$

It is stressed again that in this study the surf similarity parameter was defined using the offshore wavelenght, and the analysis was conducted separately for frequency domain and time domain parameters.

According to Rathbun et al. (1998), the data were divided into groups of $H/hs \leq 0.4$ (non-breaking waves) and $H/h > 0.4$ (breaking waves). The coefficients a and b were determined by regression analysis for the two different conditions, and are summarized in Table 2. Figures 10 (frequency domain parameters) and 11 (time domain parameters) show data and prediction method for the 2% relative runup level versus surf similarity parameter.

Table 2
Coefficients *a* and *b* for Equation 3

Frequency Domain	$Ru_{1\%}/H_{mo}$	$Ru_{2\%}/H_{mo}$	$Ru_{5\%}/H_{mo}$	$Ru_{10\%}/H_{mo}$	Ru_{sign}/H_{mo}	Ru_{mean}/H_{mo}
	$H_{mo}/h_s \leq 0.4$ (non-breaking waves)					
a	1.194	1.202	1.198	1.197	1.119	0.956
b	0.681	0.747	0.871	1.013	1.466	1.516
	$H_{mo}/h_s > 0.4$ (breaking waves)					
a	0.508	0.458	0.389	0.378	0.318	0.364
b	0.155	0.140	0.119	0.144	0.233	0.313
Time Domain	$Ru_{1\%}/H_{1/3}$	$Ru_{2\%}/H_{1/3}$	$Ru_{5\%}/H_{1/3}$	$Ru_{10\%}/H_{1/3}$	$Ru_{sign}/H_{1/3}$	$Ru_{mean}/H_{1/3}$
	$H_{1/3}/h_s \leq 0.4$ (non-breaking waves)					
a	0.664	0.614	0.530	0.460	0.310	0.264
b	0.254	0.250	0.246	0.254	0.237	0.250
	$H_{1/3}/h_s > 0.4$ (breaking waves)					
a	0.413	0.394	0.359	0.322	0.290	0.464
b	0.042	0.041	0.040	0.038	0.154	0.418

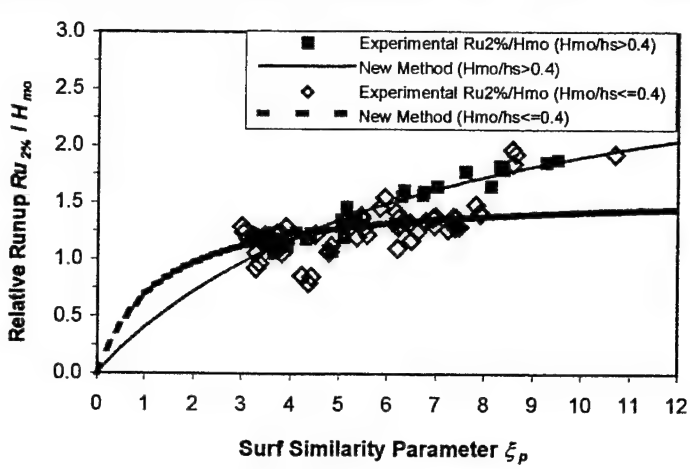

Figure 10 New Method for $Ru_{2\%}/H_{mo}$

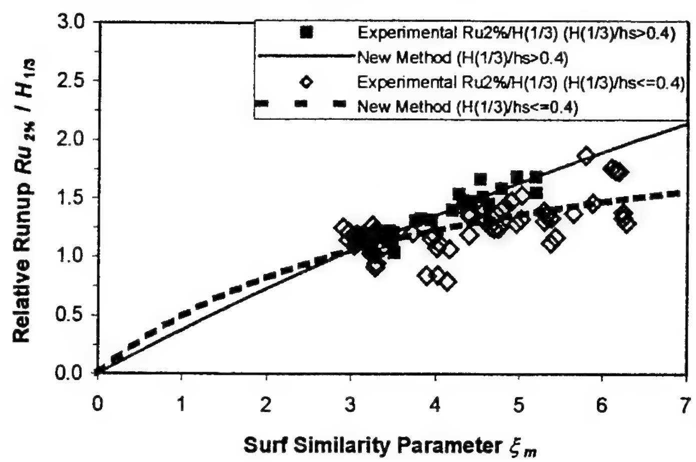

Figure 11 New Method for $Ru_{2\%}/H_{1/3}$

5. Analysis of wave transmission due to overtopping

Transmitted wave height due to overtopping was measured for a water depth of $h_s = 0.5$ m and $h_s = 0.7$ m, providing a crest freeboard of $R_c = 0.3$ m and $R_c = 0.1$ m respectively, where R_c is the vertical distance between the structure crest and the still water level. The analysis of wave transmission due to overtopping was performed using only frequency domain parameters. The transmission coefficient is defined as

$$K_t = H_t/H_i \tag{4}$$

where H_t is the transmitted significant wave height (H_{mo} measured at the location of gage 4) and H_i is the incident significant wave height (H_{mo} measured at the location of gage 6 without the structure in place).

Previous studies performed on rock armor layers (Van der Meer 1991; D'Angremond, Van der Meer and De Jong 1996; Seabrook and Hall 1998) used the dimensionless crest height, defined as R_c/H_i, to study transmission. A plot of K_t versus R_c/H_i is showed in Figure 12.

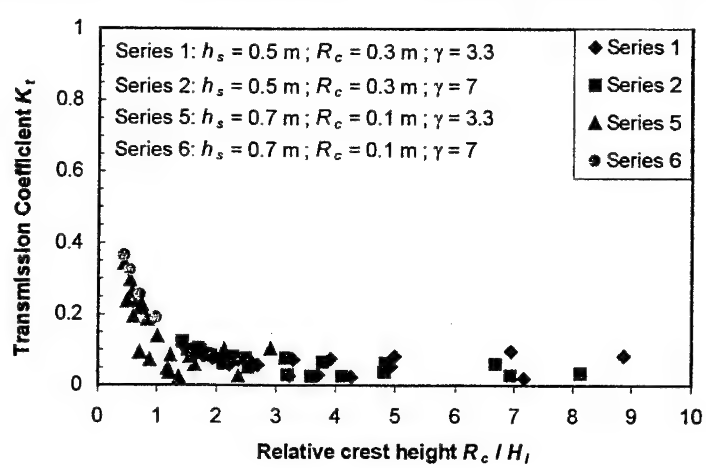

Figure 12 K_t versus R_c/H_i

It can be seen that K_t decreases as R_c/H_i increases, and a lower value of R_c/H_i (or a larger H_i) results in a higher transmission coefficient, as a larger incident wave height gives more overtopping. Moreover, for transmission, the effects of spectral peakedness are negligible.

The effects of core permeability on transmission are shown in Figure 13. In general, a medium permeability level of the core allows more transmission due to the water flowing through the more porous structure.

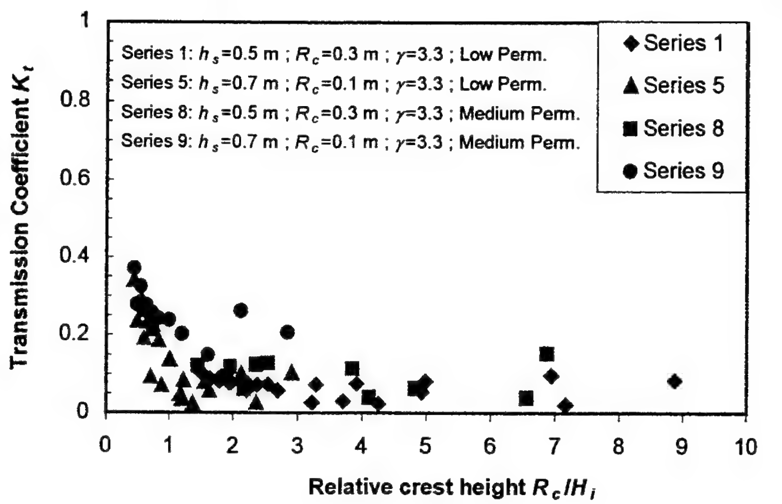

Figure 13 Effects of core permeability on K_t

Additional series of tests were performed with a submerged structure in order to get high values of the transmission coefficient. In total, 33 tests were executed. For these tests, a new structure was built in the flume with a crest height of 0.4 m and using a core of medium permeability. The crest width was again kept to three times C, the minimum for overtopping conditions, as mentioned earlier in this paper. All the tests were performed with a spectral peakedness of γ = 3.3 and a core with medium permeability. The test conditions are summarized in Table 3.

Table 3
Submerged structure: Summary of Test Conditions

Series	h_s	γ	P (Core)	No. of tests	ξ_p	H_{mo}/h_s
10	0.7 m	3.3	Medium	11	3.4 - 7.1	0.05 - 0.31
11	0.5 m	3.3	Medium	11	3.2 - 7.6	0.09 - 0.41
12	0.4 m	3.3	Medium	11	3.9 - 8.3	0.09 - 0.47

In Figure 14 is shown a plot of K_t versus the R_c/H_i for some particular wave periods including the data for the submerged structure. From this figure it can be seen the influence of the wave period. In general, for the same value of the relative crest height, wave transmission increases as wave period increases.

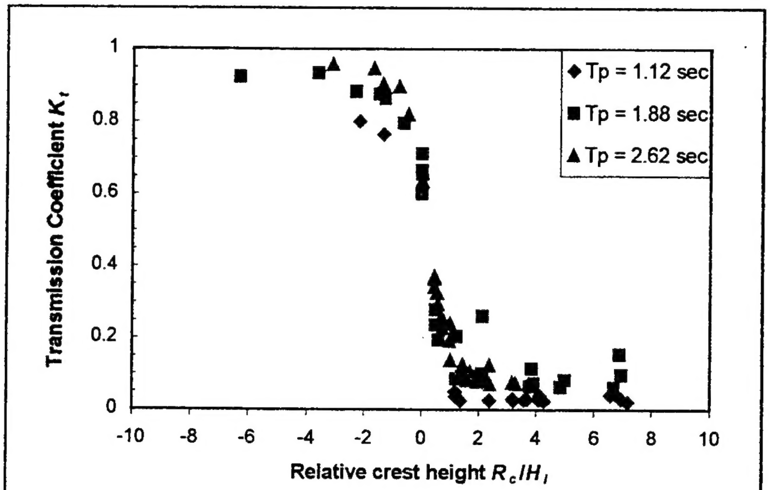

Figure 14 Effects of wave period on K_t

Plot in Figure 15 shows K_t versus the R_c/H_i for all data. A clear discontinuity in K_t values, also present in Figure 14, is evident in the region of R_c/H_i close to zero. This is due to the fact that the data sets for $R_c/H_i>0$ and $R_c/H_i \leq 0$ were obtained from structures with two different crest heights, 0.8 m and 0.4 m respectively. In the zone of negative values of R_c/H_i, a smaller absolute value of R_c/H_i (or a larger H_i) results in a lower transmission coefficient, as a larger incident wave height is more influenced by the structure crest, thus reducing transmitted wave energy.

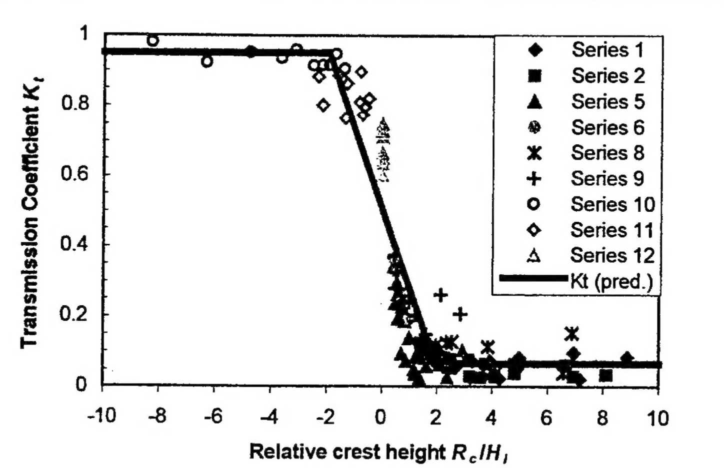

Figure 15 All data and prediction method for K_t

The crest width is an important parameter for wave transmission, but it was not taken in account for this study, as the structures were built with the minimum crest width for overtopping. It should be noted that some wave energy is lost to frictional dissipation on the surface of the structure, as waves pass over the structure (Seabrook and Hall 1998). This is a factor that may explain the scatter in the data, even if there is an influence of other factors as wave height, wave period and permeability. In general, a more porous structure will dissipate some energy as waves pass over the structure, thus reducing transmission (Seabrook and Hall 1998; D'Angremond, Van der Meer and De Jong 1996). In this case, the permeability of the armor layer, so the effects in varying the characteristic length C, should also be investigated.

The curve in Figure 15 can be used to predict K_t through R_c/H_i. The curve is described as follows:

$$K_t = 0.949 \qquad \text{for } R_c/H_i < -1.94$$
$$K_t = 0.506 - 0.228\, R_c/H_i \qquad \text{for } -1.94 < R_c/H_i < 1.94 \qquad (5)$$
$$K_t = 0.066 \qquad \text{for } R_c/H_i > 1.94$$

Even if R_c/H_i is not the only parameter that affects wave transmission as above mentioned, however this is a simple way to estimate K_t for Core-Loc armor layers.

6. Summary and Conclusions

Wave-induced runup and wave transmission due to overtopping were investigated for Core-Loc armor layers. The Core-Loc layer response was analyzed in order to evaluate the effects of wave, water level and structure conditions. The analysis of wave-induced runup was conducted separately for frequency domain and time domain parameters. A predictive empirical method (Equation 3 and Table 2) was devised from existing methods (Ahrens and Heimbaugh 1988, Rathbun et al. 1998), and fit to the data to provide directly various runup level statistics (1%, 2%, 5%, 10%, significant and mean) for non-breaking and breaking conditions. Due to the range of data used in this study, the predictive tools will be valid over the following ranges:

non-breaking waves	$3.0<\xi_p<10.7$	$3.0<\xi_m<6.3$
breaking waves	$3.7<\xi_p<9.5$	$3.2<\xi_m<5.2$

Over these ranges, the new method to predict relative runup level statistics seems relatively robust and reliable. However, further studies should be performed on wave-induced runup in order to investigate the effects of structure slopes and approaching foreshore slope.

Wave transmission due to overtopping was also investigated, but only in frequency domain parameters. Moreover, additional test series were performed on a submerged structure. A simple predictive tool for wave transmission on Core-Loc armor layers was developed from the present data. The transmission coefficient K_t was related to the relative crest height R_c/H_i as follows:

$$K_t = 0.949 \qquad \text{for } R_c/H_i < -1.94$$
$$K_t = 0.506 - 0.228\, R_c/H_i \qquad \text{for } -1.94 < R_c/H_i < 1.94$$
$$K_t = 0.066 \qquad \text{for } R_c/H_i > 1.94$$

The data on wave transmission showed some scatter, due to the fact that R_c/H_i is not the only parameter that affects wave transmission. The effects of structure crest width, armor permeability and structure slope should also be investigated in order to provide more insight on wave transmission for Core-Loc armor layers.

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